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Geometric Design of Roads

MANUAL

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CHAPTER 1

GEOMETRIC DESIGN OF ROADS

1.0 INTRODUCTION

1.0.1 Geometric design of roads embraces the visual aspects of road. It is an art based upon the work of many individuals and groups. Design methods and standards are based upon theoretical principles of geometry and physics and includes empirical results from observations and analysis of traffic behaviour, human relations, vehicle characteristics and their operations, and the desired traffic handling capacity of the road.

1.0.2 A good road design must be founded upon basic information concerning the specific project under consideration.. Such information should include:-

a) Classification of the highway facility,
b) General indication of the topography and geology of the area to be traversed by the facility,
c) Amount of funds available for the construction and subsequent maintenance.

1.0.3 The standards to be adopted for the design will be affected and influenced by the following factors:-

a) Road Classification
b) Traffic
c) Speed
d) Capacity
e) Topography and physical features
f) Vehicle Characteristics
g) Safety and Economics

1.1 ROAD CLASSIFICATION:-

1.1.1 A road network is usually grouped into a number of different classifications for administrative, planning and design purposes. Usually each class is supposed to indicate a set of minimum standards which is in keeping with the importance of the facility.

1.1.2 In Tanzania the existing classification is partly based on administrative aspects of the facility and partly on functional aspects. The existing network is basically classified in accordance with the Government Notice published in the Gazette on 5th May, 1967.

The classification takes into account and is in accordance with administrative considerations and functional role of the roads. Technical standards and quality of the roads are not incorporated in the classification system.

1.1.3 The existing classification makes distinction between the following categories of roads:-

a) Trunk roads comprising all inter-territorial and Main internal through routes. These are under the Ministry.

b) Local Main Roads, comprising all main roads other than Trunk roads.

c) Regional roads, comprising of all other roads which were funded by the Ministry.

d) District roads, comprising of classified roads other than the three categories above. These were funded by the Local Government.

e) Unclassified roads, - all other roads not covered by the above categories. These were usually funded on an adhoc basis by special agencies e.g. Cotton Authority or by local Government or villages.

1.1.4 The responsibility of constructing and maintaining all roads other than Trunk roads in now vested in the Regional administration. Some Regions have re-classified some of their roads and even eliminated some of the above classes.

1.1.5 For design purposes it is proposed that the roads be divided into 6 categories, namely:-

a) Trunk - comprising of all inter-territorial through routes.

b) Regional Roads - comprising of all inter-regional roads other than Trunk Roads and which connect Regional Centres and such centres to the Trunk Roads.

c) Regional Roads - comprising of all inter-district roads connecting district centres other than Trunk and Local Main Roads.

d) District Roads - comprising of all other major roads within a district other than the above three categories. District Roads should at least connect centres of Division (TARAFA) or main centres of production to the above main roads

e) Major Feeder Roads - comprising of all roads other than the 4 categories above which connects Wards (KATA).

f) Minor Feeder Roads - comprising of all other motorable roads.

1.1.6 Minimum design criteria based on the classification under 1.1.5 is as shown in table 1.1

Table 1.1: Minimum Design Criteria

Road Class	Carriageway Width (m)	Shoulders (m)	Bridge Loading	Right-of-way Width (m)
Trunk	6.5	1.5	HA or HS20	45
Local Main	6.5	1.5	- do -	45
Regional	6.0	1.5	- do -	30

District	5.5.	1.5	H 15	20
Major Feeder	4.5	1.2	H 15	15
Minor Feeder	3.5	1.2	H 15	10

Note:- HA loading refers to BS 153 HA loading
HS20 loading refers to AASHO HS20-40 loading
H 15 loading refers to AASHO HS15-44 loading.

1.2 TRAFFIC:

1.2.1 The design of a road should be based on the estimated traffic expected to use the facility. A good design should provide flexibility for changing conditions. A period of 10 - 20 years is usually used as the basis of traffic prediction.

1.2.2 Apart from volume, the number of each of the different types of vehicles should be estimated. Usually the traffic date should at least distinguish between heavy and light vehicles. Light vehicles are those of less than 1500 kg. unladen weight.

1.3 SPEED:

1.3.1 The design speed is used for the correlation of the geometric characteristics and features of the road. It is dependent upon the type of terrain, the type and volume of traffic and the type of road to be constructed. It is the maximum safe speed that can be maintained over a specified section of a road when conditions are so favourable that the design features of the road govern.

1.3.2 All elements of a design should be based on the same design speed which should be a good approximation of the actual operation speed.

1.3.3 The recommended minimum design speeds for the various carriageway widths are as shown in table 1.3

Table 1.2 Minimum Design Speeds

Carriageway (m)	Design Speed (km/h)		
	F/R	R/H	M
3.5	60	40	30
4.5	65	50	30
5.5	65	50	40
6.0	65	50	40
6.5	80	65	50
7.0	100	80	60

Note: F/R - Flat to rolling terrain
R/H - Rolling to hilly terrain
M - Mountainous terrain

1.4 CAPACITY AND OTHER FACTORS

1.4.1 Capacity:-

1.4.1.1 The capacity of a two -lane road is the maximum number of vehicles that can pass over a given section of a roadway in both direction over a given time period, usually one hour. It is the maximum rate of flow that has a reasonable expectation of occuring.

The capacity of a road is dependent upon the level of service the facility is intended to provide. A level of service is a qualitative measure of the effect of traffic flow factors, such as speed and travel time, interruptions, freedom of maneuver, drivers comfort and convenience, and, indirectly, safety and operational costs.

For traffic analysis purposes and especially for capacity measurements, all traffic volumes are expressed in terms of passenger car units. The basic unit is the car which is considered as equal to 1 pcu. Equivalent factors for other types of traffic is as follows:-

Type of Traffic	Urban Road	Rural Roads
Light vehicles of up to 1.5T un-ladenweight	- 1.00 pcu	1.00 pcu
Motor Cycles	- 0.75 pcu	1.00 pcu
Truck of more than 1.5T un-laden weight	- 2.00 pcu	3.00 pcu
Buses	- 3.00 pcu	3.00 pcu
Pedal Cycles	- 0.33 pcu	0.50 pcu

1.4.1.4 The capacity and maximum service volume of a roadway is affected by a number of factors. These are:-

a) Roadway factors which include:-
   i) Carriageway and shoulder width
   ii) Alignment and sight distances (passing)
   iii) Surface condition
   iv) Grades
   v) Intersections
   vi) Obstructions (lateral).

b) Traffic flow factors such as:-
   i) Percentage of heavy vehicles
   ii) Traffic interruptions such as pedestrian crossing and parked vehicles
   iii) Speed range found within the traffic stream
   iv) Number of overtaking and passing maneuvers required to maintain a desired speed.

C) Environmental factors such as weather

1.4.2 Topography and physical features

1.4.2.1 For design purposes topography is classified into three terrain categories, namely:-

a) Flat to Rolling terrain
b) Rolling to Hilly terrain
c) Mountainous terrain

1.4.2.2 In level terrain the cross slope is usually less than 10%. Where the slope is between 10 and 25% such a terrain is called rolling. The cross slope in hilly terrain is of the order of 25 to 60%. In mountainous terrain the cross slope should be greater than 60%.

1.4.3 Vehicle Characteristics

1.4.3.1 The physical characteristics of vehicles and their relative percentages (traffic composition) of various sized vehicles anticipated to use the facility must be considered in geometric design Usually traffic is categorized into two main groups:-

a) Light vehicles which should include passenger cars, pickups, light trucks (of less than 1.5T unladen weight).
b) Heavy vehicles which should include buses, single unit lorries, semi-trailers and truck-trailers.

1.4.3.2 A design vehicle must also be chosen. This is the vehicle whose weight, dimensions and operational characteristics will be used in highway design, i.e. the design should accommodate such a vehicle.

1.4.4 Safety and Economics

1.4.4.1 Decisions as to the exact location and details of geometric design must be based on cost-benefit analysis that takes into account all factors concerned. The purpose of the analysisshould be to determine whether or not the maximum benefits to be provided by the road are consistent with the costs involved.

1.4.4.2 The costs to be considered should include:-

a) Total construction cost including design and supervision costs.

b) Total maintenance cost over the entire design life of the road.

1.4.4.3 The benefits to be evaluated should include:-

a) Saving in vehicle operating costs

b) Saving in time

c) Others such as increased safety, comfort etc.

All values of cost and benefits must be evaluated in money terms and based on a common (base) year.

For most designs the cost - benefit analysis is based on saving in time and vehilce operating cost only. The other benefits are evaluated in qualitative manner only.

1.4.4.4. Safety is enhanced on highway by proper design controls and enforcement of the controls. The design can enhance safety if proper design speeds are used and there are no sudden changes in the alignment or the geometrics.
Proper traffic signing and markings is also a must in this aspect.
To get road users to obey the operational rules of the road, they must be educated and all offenders must be prosecuted other-wise they might adopt the "I can-get away" attitude.

1.5 GEOMETRIC DESIGN STANDARDS

1.5.1 The geometric design standards for bitumen, gravel/earth and un-laned roads (one-lane two-way carriageway) are given under paragraphs 1.6, 1.7 and 1.8 respectively. Those for elements not covered under these paragraphs are given below.

1.5.2 Carriageway width on bridges.

On two-lane bridges the carriageway width (kerb to kerb) should be at least 1.0m wider than that of the approaches. For all Trunk roads the minimum width should be 7.5m.

On single-lane bridges the minimum width should be 4.0m. Where footpaths are provided, their minimum width should be 1.0m, if provided on both sides, and 1.5m if provided only on one side.

1.5.3 Camber

Adequate camber (cross-slope) should be provided on the roadway to facilitate rapid run-off for water falling on the road surface.

- Asphaltic concrete surfaces - 2.0 to 2.5%
- Surface dressing surfaces - 2.5%
- Stone paved surfaces - 3.0%
- Gravel and earth surfaces - 3.0%

Shoulder Slopes should normally be 0 to 3% steeper than that of the carriageway, depending on rainfull, construction materials and erosion control measures adopted.

1.5.4 Side Slopes

Side slopes should be as flat as economically feasible. Recommended maximum rates of side slopes for embankment (fills) are as follows:-

Embankment Height (m)	Side Slope (vertical: Horizontal)
Less than 1.0m	1:4
1.0 to 3.0m	1:2
Greater than 3.0m	1:1.5

1.5.5 Back Slopes

Recommedend maximum rates for back slope are as follows:

Hard Rock	- 1:1 to 4:1
Decomposed Rock and/ compact lateritic soils	- 1:2 to 1:1
Ordinary soils	- 1:2 to 1:1.5

Benching should be done at 3.0 to 5.0m intervals. The actual maximum rate of slope should be based on local experience.

1.5.5.1 Minimum height of side drains should be

To shoulder	0.5m
To subbase	0.2m

1.5.5.2 The free height from top of pavement to underside of structures should be greater or equal to 4.5m.

1.5.5.3 The maximum gradient of longitudinal alignment should be

Mountainous terrain

Hilly terrain

Flat/Rolling terrain

1.5.6 Sight Distances

1.5.6.1 Sight distance is the continuous length of road ahead visible to the driver. The length required for different vehicle maneuver will depend on the nature of the maneuver, vehicle speed and condition of the road.

1.5.6.2 The distance required for stopping is called the stopping sight distance. Stopping sight distance is the distance required for safe stopping operation where opposing-direction traffic is not involved in passing operation. The minimum distance should be sufficient at the design speed for perception and brake-reaction time and for bringing the vehicle to stop.

1.5.6.3 Passing sight distance is the shortest distance sufficient for a vehicle to turn out of a traffic lane, pass another vehicle, and return back to the same lane safely and comfortably; without interfering with the overtaken vehicle or an oncoming vehicle travelling at the design speed, should it come into view after the passing maneuver is started.

1.5.6.4 Sight Distance on Horizontal Curves
In this case the sight line is a chord and the sight distance is measured along the centreline of the inside lane of the curve.

Height criteria do not affect design for horizontal curves to fit minimum stopping sight distance requirements except where the obstruction is a cut slope. In that case the following values should be used:
Height of eye - 1.15m
Height of object -0.15m

A height of 0.6m can be used to approximate the midpoint of the sight line where a cut slope on the inside of curve, obstructs the line of sight. The critical point on a cut is a point 0.6m above the road (pavement)level for stopping sight distance checking.

1.5.7 Curve Widening
Where transition curves are provided, widening may be placed on the inside or divided equally between the outside and inside of the circular curve. On curves which have no transition, the widening should be applied on the inside edge of the pavement only. Regardless of how widening is effected, the final centreline markings should be placed Midway between the edges of the widened pavement.

1.6 DESIGN STANDARDS FOR BITUMEN ROADS

1.6.1 Design Speed:

Desirable and minimum design speeds are given below:- Higher design speeds should be used so long as they are economically feasible and are consistent with the carriageway width adopted.
Caution: High design speeds need wider carriageway for satisfactory performance.

Table 1.3 Design Speeds

Current ADT (vpd)	Carriageway Width (m)	Design Speed (km/h)					
		Desirable			Minimum		
		F/R	R/H	M	F/R	R/H	M
550-150	6.0	80	65	50	65	50	40
150-400	6.5	100	80	60	80	65	50
400-1000	7.0	110	80	65	100	80	60
Over1000	7.5	110	80	65	100	80	60

F/R = Flat to Rolling terrain
R/H = Rolling to Hilly terrain
M = Mountainous terrain.

1.6.2 Roadway width

Carriageway widths and corresponding shoulder widths are given below. The shoulder width given below are to be considered as minimum recommended values. Wides shoulder should be adopted whenever it is economically feasible.

Carriageway width (m)	6.0	6.5	7.0	7.5
shoulder width (m):	1.5	1.5	2.0	2.0

1.6.3 Roadway transverse slopes

The carriageway slope should be as follows:
Normal camber - Asphaltic concrete - 2.0% to 2.5%
- Bitumen Surfaces treated - 2.5%
The shoulder slope should be at least equal to carriageway slope and not more than 3% greater. The lower limit is for paved shoulder and the higher for grassed, gravel or earth shoulders.
See also para 1.5.3

1.6.4 Minimum Radius of Curve

Desirable and absolute minimum radii of curves are given below for each design speed and maximum superelevation rate (e_{max}).

Table 1.4 Minimum Radius of Curves

Design Speed km/h	Minimum Radii of Curves (m)		
	Desirable	Absolute	
		e_{max} = 0.08	e_{max} = 0.06
120	750	600	670
110	600	480	530
100	450	380	420
80	300	230	250
65	200	145	160
60	150	125	135
50	100	85	90
40	75	50	55
30	50	30	30

1.6.5 Superelevation

1.6.5.1 The rates of superlevation to be applied can be calculated using the followjng relationship:-

$e = \frac{V^2}{314R}$ where e = superalevation rate (fraction)

V = design speed (km/h)

R = radius of curve (m)

The values of superalavition to be applied should be rounded - off to the nearest 0.001 or 0.1%

The maximum rate of supperalevation should be 0.08 or (8%) and 0.06 (6%) for Flat to Hilly and Mountainous terrain respectively.

1.6.5.2 The Minimum rate of superelevation should be 0.01 or (1%).

1.6.5.3 It is recommended that for calculated rates of superelevation between the normal cross-fall (0.025 or 0.020) and 0.015, the superlevation rate to be applied should be that equal to the normal cross-fall i.e 0.025 or 0.020. For rates of superelevation between 0.01 and 0.015, only adverse camber need to be replaced by favourable slope. The corresponding controlling curvature values are given below.

Table 1.5: Superelevation Limits

Design Speed (km/h)	Minimum radius (m) for:-			
	Normal Camber	Removal of Adverse Camber	Superelevation rate equal to normal crossfal	
			e = 0.020	e = 0.025
120	4600	3100	2250-3100	1800-3100
110	3900	2600	1880-2600	1520-2600
100	3200	2200	1560-2200	1250-2200
80	2100	1400	1000-1400	800-1400
65	1400	900	660-900	530-900
60	1200	800	560-800	450-800
50	800	550	390-550	310-550
40	500	350	250-350	200-300
30	300	250	140-200	113-200

Superelevation run-off length

Where transition curves are provided the superelevation should be effected throughout the length of such curve. The length required to change from normal camber to a superelevation rate equal to the normal cross-fall (slope at normal camber) i.e from - 0.02 and - 0.025 to + 0.025 for asphaltic concrete and bitumen treated surfacing respectively, are given in Table 1.4 below. If the superelevation required is greater than that of the normal cross-fall (absolute value), then the balance of the superelevation should be effected along the ramaining transition length, at a uniform rate.

For example

For a 1200m radius curve, for 110 km/h design speed the required rate of superelevation is 0.032 or 3.2%. If the normal cross-fall is 0.025 or 2.5% and carriageway width is 6.5m the required length of transition curve will be 133m. The required length to change from normal camber to a superelevation rate of 2.5% is 42m (see table 1.4 below). Thus, the distance needed to change from normal camber to a superelevation rate of 2.5% and to change from e - 2.5% to 3.2% will be 42m and 133-42 = 91m respectively.

The two changes should be effected uniformaly over the two corresponding distances. If the radius of the curve was 1540m, the required superelevation rate could have been 2.5% and the transition length needed would have been 171m. In this case the change from normal camber to the superelevation rate of 2.5% should have been effected along the first 42m of the transition curve. The rest of the transition curve (129m) would have been maintained at 2.5% throughout.

Where no transiton curve is provided superelevation run-off length should be assumed.
The length of such run-off length is given by:-

$$L = \frac{e - e_o}{E} \cdot \frac{C}{2}$$

where $e = \frac{V^2}{314R}$. $\quad e = \frac{V^2}{314R}$

E = rate of change of superelevation as given in table 1.4. below.

e_o = normal camber slope (-0.02 or 0.025)

C = carriageway width (m)

Two-thirds of the superelevation run-off length so calculated should be outside the circular curve (on tangents) and one-third on the curve.

Table 1.6 Superelevation run-off lengths needed to change from normal camber to superelevation rate equal to the normal camber.

Design Speed / Carriage-way length	Superelevation run-off length (m)								Rate of change of super-elevation E (%)
	CHANGE IN (C) -2.0% to +2.0				Change in e from -2.5% to +2.5%				
	6.0	6.5	7.0	7.5	6.0	6.5	7.0	7.5	
120	33	33	36	39	39	42	45	48	0.38
110	30	33	36	36	36	42	45	48	0.40
100	27	30	36	36	36	39	42	45	0.43
30	24	27	27	30	30	33	36	39	0.50
25	21	24	24	27	27	30	30	33	0.57
60	21	21	24	24	24	27	30	30	0.60
50	18	21	21	24	24	24	27	27	0.66
40	18	18	18	21	21	24	24	27	0.72
30	15	18	18	18	18	21	21	24	0.78
Marks	Asphaltic Concrete				Bitumen Treated				

1.6.6 Transition curve Lengths

The clothoid curve is to be used as the transition. The equation of the clothoid is:-

$L = \frac{A^2}{R}$ Where A = clothoid parameter

R = radius of circular curve

To estimate the length of transition curve needed, three criteria are to be used, and the higher value is to be adopted.

These criteria are:-

(a) Rate of change of centipetal acceleration (q). For comfort the rate of the change of centripetal acceleration should be 0.5m/sec^3 and 0.6m/Sec^3 for design speeds greater and equal or less than 50km/h, respectively.

To meet this criterion the transition length (L) should be:-

$L = \frac{V^3}{23.5R}$ for design speeds greater than 50km/h.

$L = \frac{V^3}{28R}$ for design speeds equal or less than 50km/h.

(b) Superelevation run-off contained within the transition: The length of the transition curve should be at least equal to the required superelevation run-off length. To meet this criterion, the length of the transition(L) should be

$L = \frac{e - e_o}{2} \quad \frac{C}{2}$ Where e_0 = -0.02 or -0.025

E = rate of change of superelevation (see table 1.4)

C - carriageway width (m)

$e = \frac{V^2}{314R}$

(c) AESTHETICS

In order to see the characteristic shape of the clothoid, it should consume and angle of at least 3^0. This condition is met if the length of the transition (L) is:-

$L = \frac{R}{9}$

Curves of larger radius than the values given in in Table 1.7 may be used without transition curves.

Table 1.7 Minimum Radius for no transition

Design Speed (km/h)	Minimum radius (m)	
	e_0 = 0.020	e = 0.025
120	2300	1900
110	2000	1600
100	1600	1300
80	1100	900
65	700	600
60	600	500
50	400	350
40	250	200
30	150	120

1.6.7 Limiting grades:

Recommended maximum grades are given in table 1.8 below. For low volume rural roads and /or short lengths of grades (less than 150m) the value in the table can be increased by 1%.

Minimum grades on cut sections should usually be 0.5% unless special drainage treatment are provided.

Table 1.8 Maximam Grades (%)

Terrain	Design Speed (km/h)								
	30	40	50	60	65	80	100	110	120
Flat/Rolling	6	6	5	5	5	4	3	3	3
Rolling/Hilly	8	8	7	6	6	5	4	4	4
Mountainous	10	10	9	8	8	7	6	5	5

1.6.8 Critical Length of grade is the maximum length of the designated up-grade on which a loaded truck can operate without unreasonable reduction in speed. As such the length of all grades should be limited to the values given in table 1.9 for a 24 km/h reduction in speeds of trucks.

Table 1.9: Critical length of Grades

Grade	Critical length (m)	Maximum sustained speed km/h
2.0	850 (1100)	40
2.5	640 (850)	-
3.0	520 (670)	25
3.5	410 (540)	-
4.0	340 (445)	16
4.5	275 (370)	-
5.0	235 (320)	15
5.5	210 (275)	-
6.0	185 (245)	13
6.5	170 (225)	-
7.0	160 (210)	12
7.5 and over	150 (190)	-

The values given in table 1.9 are based on a 24 km/h reduction in speed of trucks (from the average running speed). The corresponding values in brackets are for 40km/h reduction in speed.

The values for 40km/h reduction in speed are given here so that one can get an indication of the effect of grades longer than critical. These critical lengths are also based on tangent lengths.

Where a vertical curve is part of a critical length, an approximation must be made as to the equivalent tangent grade length. If the algebraic difference in grade is small, the measurement of critical length of grade may be made between the V.I.P.'s. If the algebraic difference is appreciable, one quarter(1/4)of the vertical curve length may be considered as part of the critical grade.

The maximum sustained speed is the top speed trucks can maintain on a given up-grade for lengths greater than critical i.e. the constant speed. It is assumed that the truck in question has a weight power ration of 400.

1.6.9 Sight distances:

Manimum stopping and passing sight distances are given in Table 1.8 below. The minimum sight distance are for both horizontal and vertical alignment (curves)

Table 1.10:- Minimum Sight Distances (m)

Design Speed (km/h)	Stopping Sight Distances (m)	Passing sight Distances (m)
120	210	790
110	185	760
100	160	680
80	115	550
65	85	440
60	75	400
50	55	330
40	40	250
30	30	180

1.6.10 Length of Vertical Curves

Recommended minimum length of vertical curves are given in Table 1.11 Where the recommended length is greater than 43.5A special attention should be given to drainage.

Where kerbs are used it may be necessary to remove them so as to provide adequate transverse drainage.

Table 1.11: Recommended Minimum length of Vertical Curves (m)

Design Speed km/h	To satisfy stopping sight distanance(m)		To satisty passing sight distances(m)
	Crest	Sag	
120	103A	55A	660A
110	80A	45A	610A
100	60A	40A	495A
80	32A	25A	325A
65	17A	17A	205A
60	14A	16A	170A
50	10A	12A	115A
40	6A	8A	70A
30	4A	6A	35A

Note: A = algebraic difference in grade in percent %
i.e for A = 10%, in the table should be 10.

1.6.11 Curve Widening

The carriageway width given in Table 1.12 below are for a 7.0m carriageway. For other carriageway widths, the value in the table should be modified by the following factors:

Carriageway width (m)	Adjustment factor
6.0	Subtract 0.50m
6.5	" 0.25m
7.0	No change
7.5	Add 0.25

Example:

For a 200m radius curve, design speed = 80 km/h the width required at the curve is 7.65m for a 7.0m carriageway. For same curve and design speed, the width needed for 6.0, 6.5 and 7.5m carriageway is 7.15, 7.40 and 7.90 respectively.

Once the required carriageway width is known, the additional widening needed is the difference between the required width and the carriageway width.

Thus for the example given above the widening needed is 1.15, 0.90, 0.65 and 0.40m for 6.0, 6.5, 7.0, and 7.5m carriageway respectively. Widening values less than 0.5m, widening should be provided and the amount of widering rounded off to the nearest 0.25m for practical purposes.

Table 1.13 below gives such values for a 6.5m carriageway.

Table 1.12: Width of roadway neede d at curve for a 7.0m carriageway

Curve Radius (m)	Design Speed (km/h)								
	30	40	50	60	65	80	100	110	120
30	8.95								
40	8.49								
50	8.21	8.35	8.50	8.65	8.75	8.95	9.25	9.40	9.55
60	8.02	8.15	8.30	8.45	8.50	8.70	9.00	9.10	9.25
80	7.78	7.90	8.00	8.15	8.20	8.35	8.60	8.70	8.86
100	7.63	7.75	7.85	7.95	8.00	8.15	8.35	8.45	8.55
150	7.42	7.50	7.60	7.65	7.70	7.85	8.00	8.10	8.20
200	7.30	7.40	7.45	7.55	7.55	7.65	7.80	7.90	7.95
250	7.24	7.35	7.45	7.45	7.45	7.55	7.70	7.75	7.85
300	7.19	7.25	7.30	7.35	7.45	7.50	7.50	7.65	7.75
400	7.13	7.20	7.25	7.30	7.30	7.40	7.50	7.55	7.60
500	7.09	7.15	7.20	7.25	7.25	7.30	7.40	7.45	7.50
600	7.06	7.10	7.15	7.20	7.20	7.25	7.35	7.40	7.45
800	7.03	7.05	7.10	7.15	7.15	7.20	7.30	7.30	7.35
1000	7.00	7.05	7.05	7.10	7.10	7.15	7.25	7.25	7.30
1200	7.00	7.00	7.05	7.05	7.10	7.15	7.20	7.25	7.25

Note :

i) The values given in the table are the required carriageway widths at curves of various curvatures and design speeds, for a 7.0m carriageway.

ii) To get similar values for other carriageway widths other than 7.0m, the following procedure must be used.

a) Compute the difference in width between the carriageway width is question and 7.0m.

b) Add or subtract half of the difference in carriageway width to the coresponding values in the table for carriageway greater or less than 7.0m respectively.
e.g. R = 200m design speed 80km/h. Width needed at curve for 6.0m carriageway
= 7.65 - ½ (7.0 - 6.0)m
= 7.15m
For the same curve for 7.5m carriageway the width will be 7.90m
For design purposes, the widening to be applied must be at least 0.5m and usually estimated to the nearest 0.25.
For the examples given above, widening needed in as follows:-
6m Carriageway = 7.15 - 6.0 = 1.15 say 1.25m
7.5m Carriageway = 7.90 - 7.5 = 0.40 i.e no widening needed.

Table 1.13 Widening needed for 6.5m Carriageway

Curve Radius (m)	Design Speed (km/h)								
	30	40	50	60	65	80	100	110	120
30	2.25								
40	1.75								
50	1.50	1.50	1.75						
60	1.25	1.50	1.50						
80	1.00	1.25	1.25	1.50					
100	1.00	1.00	1.00	1.25	1.25				
150	0.75	0.75	0.75	1.00	1.00				
200	0.50	0.75	0.75	0.75	0.75	1.00			
250		0.50	0.50	0.75	0.75	0.75			
300		0.50	0.50	0.50	0.75	0.75	0.75	0.75	
400 500			0.50	0.50 0.50	0.50 0.50	0.75 0.50	0.75 0.50	0.75 0.75	 0.75
600						0.50	0.50	0.75	0.75
800							0.50	0.50	0.50
1000							0.50	0.50	0.50
1200								0.50	0.50

1.6.12 Sight Distance on Horinzontal Curves

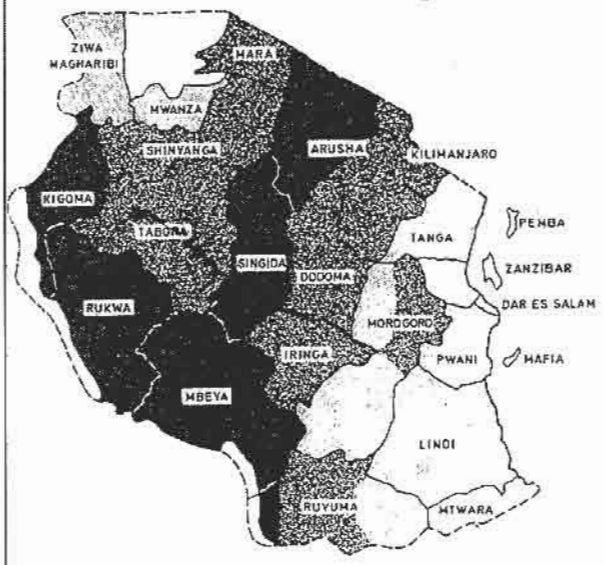
The sight line for this purpose is a chord and the sight distance is measured along the centerline of the inside lane around the curve.

The minimum distance from the centerline of inside lane to an obstruction (lateral) are given in Table 1.12 below. The Distance given satisfies stopping sight distance requirements.

Table 1.12: Minimum distance from the centerline of inside lane to an obstruction

Radius (m)	Distance (metres) for various design speeds (km/h)								
	30	40	50	60	65	80	100	110	120
30	3.7								
40	2.8	4.9							
50	2.2	3.9							
60		3.3	6.2						
70		2.8	5.3						
80			4.7						
100			3.8	7.0					
110			3.4	6.3	8.1				
130			2.9	5.4	6.9				
150				4.7	6.0				
200				3.5	4.5	8.2			
250					3.6	6.6			
300					3.6	5.5	10.6		
350						4.7	9.1		
400						4.1	8.0	10.6	
450							7.1	9.5	
500							6.4	8.5	11.0
600							5.3	7.1	9.2
700							4.6	6.1	7.9
800							4.0	5.3	6.9
1000								4.3	5.5
1200								3.6	4.6
1500									3.7

From BRU: Technical Guideline No. 2
(Loads for Structural Design)



	RISK LEVEL	RICHTER SCALE	k_a*
ZONE 0	Rare, unknown	0 - 4	0,0
ZONE 1	Week, not frequent	3 - 5	0,025
ZONE 2	Not violent, moderately frequent	5 - 6	0,050
ZONE 3	Strong, frequent	6 - 7	0,075

*) k_a = Ground acceleration factor

Fig. SEISMIC RISK MAP OF TANZANIA

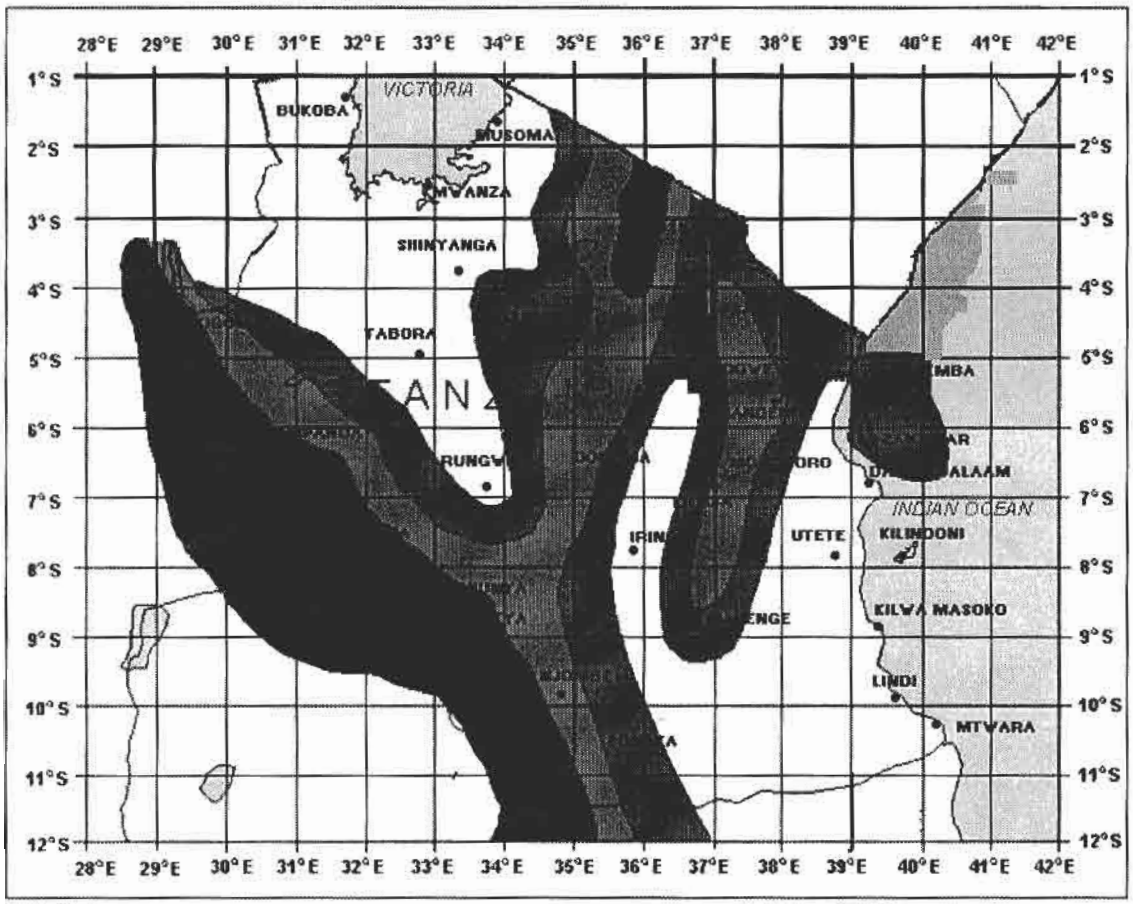
Fig. . Seismic risk map of Tanzania

Table 5.3.3. The horizontal force factor k_n. The factor reflects the manner in which structures resist earthquake forces and the ductility of the material used

Description of structure	k_n
Building with a ductile* moment resisting space frame with capacity to resist the total lateral loads independently	0.7
Building with a ductile moment resisting space frame with capacity to resist at least 25% of the total lateral loads. The building should in addition consist of shear walls with a total capacity to resist the total lateral loads independently of the space frame	0.8
Building with ductile framing or ductile shear walls different from other structures described elsewhere in this table with a sufficient capacity of the framing or walls to resist the lateral loads	1.0
Building with shear walls consisting of reinforced concrete, reinforced masonry or structural steel, a combination of these materials	1.3
Building of unreinforced masonry	2.0
Elevated tanks or stores (with contents) with crossbraced legs	3.0

*Steel and reinforced concrete (not pre-stressed) may be regarded as ductile materials

EARTHQUAKE AREAS



KEY:

- High degree of risk
- Fairly high degree of risk
- Slight degree of risk
- Low degree of risk

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